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EVALUATION OF STRUCTURAL WALLS DESIGNED ACCORDING TO EUROCODE 8 AND SIA 160

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SUMMARY

For earthquake action the new design provisions of Eurocode 8 are in the process of replacing the European national earthquake codes. The paper treats the design and behaviour of multi-storey structural walls in view of the new provisions. For structural walls the provisions of the Eurocode 8 are compared with those of a national code which it is going to replace. As the national code the current Swiss earthquake standard SIA 160 is chosen. Basic design rules of both codes are introduced and compared by means of examples comprising buildings which are regular in plan and elevation and which use structural walls for lateral resistance. The height of the buildings is varied from a from four to eight storeys. In the example, both the SIA and the Eurocode design provisions are based on the static equivalent force method, and a triangular distribution of the lateral force. However, most other design provisions differ between the two codes. The structures designed are modelled numerically and subjected to non-linear time-history analysis. At first, both the SIA and Eurocode designed structures are subjected to ground motions compatible to the design spectra in the respective codes. Then all structures are subjected to a recorded ground motion. The results are discussed in view of assumptions made at the design phase. Conclusions and recommendations are provided. © 1998 John Wiley & Sons, Ltd.

KEY WORDS: Eurocode; non-linear dynamic analysis; numerical models; reinforced concrete; seismic codes; structural walls

INTRODUCTION

General

Designs of earthquake-resisting reinforced concrete structural walls have evolved through the past 20 years to comprise principles established in the capacity design method. This method, defined by Paulay *et al.*,[1] Paulay and Priestley,[2] and Bachmann,[3] attempts to achieve ductile and predominantly flexural behaviour in the non-linear range. The shear behaviour should remain within the elastic range since failure in shear occurs suddenly and may cause a structural collapse.

The European design provisions for design of earthquake-resisting structures, Eurocode 8, henceforth referred to as EC 8, published in 1995 as a European pre-norm,[4] to be adopted in a final version during 1998, embraces for the first time in Europe the principles of capacity design. This applies also to the design provisions of structural walls, which are divided into different ductility classes. A magnification factor for shear is introduced which attempts to prevent a premature shear failure.

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In a recent study Wenk and Bachmann[8] compared two EC 8 wall designs with a corresponding wall designed according to the Swiss National standard SIA 160.[5] Among their findings are: more reinforcement may be needed, and larger wall thickness may be required when designing according to EC 8 compared to SIA.

Until now, the non-linear dynamic behaviour of a structural wall building designed according to EC 8 has not been investigated and viewed in relation to the design objectives, to the knowledge of the author. This study attempts to fill an urgent need for an evaluation of the design provisions of this new code.

Objective

This paper focuses on the design rules and the behaviour of structural wall buildings designed according to EC 8 and a national code which it is replacing: SIA 160. Based upon the yet unfulfilled needs identified above, the objectives of the study are summarized as follows.

(1) Based on examples, involving four-storey and eight-storey reinforced concrete buildings, regular in plan and elevation and with structural walls for seismic resistance, introduce the basic design provisions of both codes.
(2) By means of a non-linear numerical model, developed for realistic yet efficient simulation of non-linear behaviour of structural walls, carry out a non-linear time-integration analysis for the walls, using three different ground acceleration histories.
(3) Relate the results from the numerical analyses to assumptions made at the design phase for both codes using ground motions compatible to the design spectrum used, and to compare the predicted relative behaviour of the walls.
(4) Provide conclusions and recommendations for future studies and propose improvements for the design rules of the studied codes.

Scope

Based on the objectives given above, the scope of this study is summarized as follows. In the subsequent section the design of walls according to the two codes is described. In the next section the numerical model and the analysis is presented. The results of the analysis are discussed in the following section. Finally, conclusions and recommendations focusing on practical design are provided.

DESIGN OF STRUCTURAL WALLS

As a design example the building introduced by Linde[9] and shown in Figure 1 is used. This building comprises eight storeys of three by five bays. A version with four storeys, seen in the figure, is also used in the subsequent examples, all dimensions remaining the same as for the eight-storey building for simplicity. The buildings employ reinforced concrete structural walls for lateral resistance. Reinforced concrete gravity load columns carry all gravity loads transferred by the reinforced concrete slabs supported by T-beams. The columns and T-beams are only designed to carry gravity loads and are not resisting earthquake actions. For simplicity, no basement storey is included, and the walls are assumed to be well anchored into a stiff foundation slab. Materials used are summarized in Figure 1.

Due to symmetry only one of the two walls in the short direction will be analyzed, to which half of the lateral storey masses are attributed in the following static equivalent force calculation. The storey weights are obtained[9] as follows: 2·78 MN for the roof, and 3·08 MN per floor for all intermediate floors. The design procedure will be treated separately for SIA and for Eurocode, beginning with SIA 160. For a more detailed discussion on the steps involved in the design procedure, see e.g. Linde and Moehle.[15]

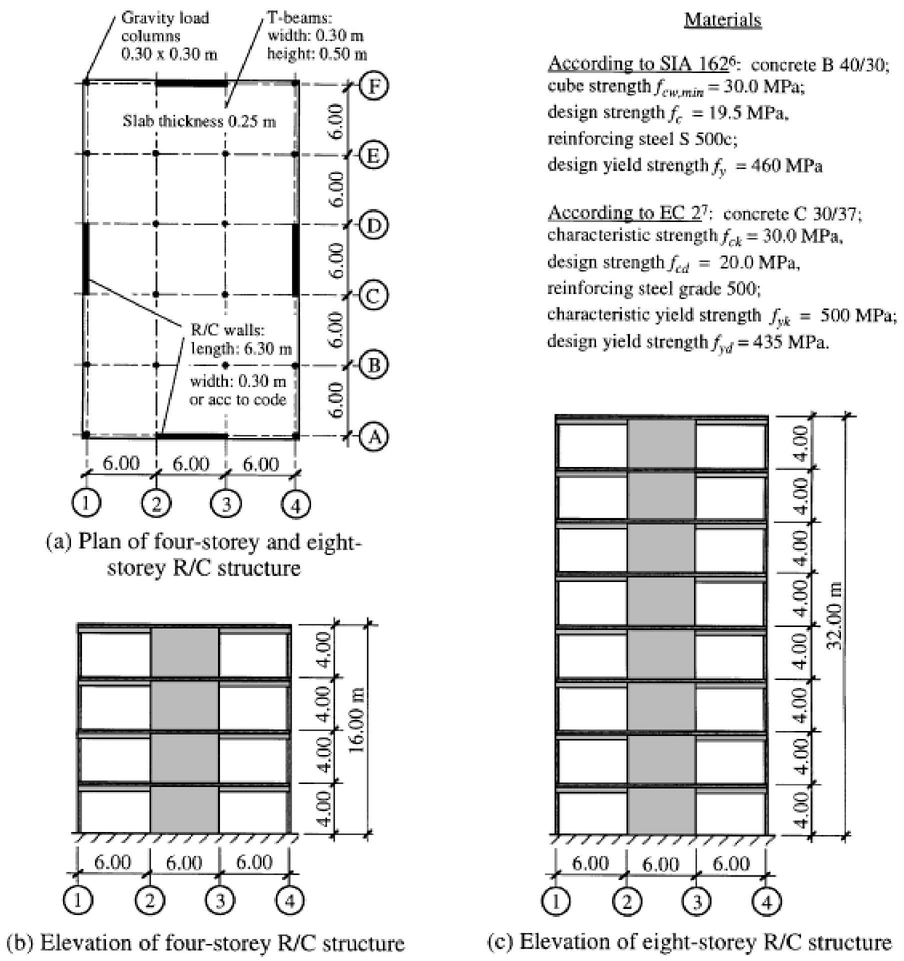


Figure 1. Four-storey and eight-storey reinforced concrete structures: dimensions and materials

Design according to SIA 160

The design of the structural wall for earthquake action is based on a lateral static equivalent force. The force consists of a fraction of the building weight multiplied by a design response spectrum acceleration value. This acceleration value is obtained at a frequency corresponding to the fundamental frequency of the building as given in the code. The building is placed in structural class 1, SCI, the lowest of three classes of increasing importance and verification requirements of SIA.

The design procedure will be illustrated using the eight-storey structure. For the fundamental frequency of the building the code proposes a formula, which may be used. Other calculation methods may be used instead of the formula. In this study, the fundamental period $T_1 = 1{\cdot}27$ s, obtained by the numerical model (see subsequent section) is used. For comparison, the code formula delivers a period $T_1 = 0{\cdot}65$ s, which is a low value, and which is deliberately taking non-structural elements into account. A wall designed based on the code formula's frequency was analysed numerically and did not reach inelastic behaviour. Its results are not included in the subsequent discussion.

Using the SIA design response spectrum in Figure 2, assuming zone 3b, i.e. the zone of the highest seismicity, and medium stiff soil, $T_1 = 1{\cdot}27$ s results in a_h/g of 0·135. The total weight is obtained as

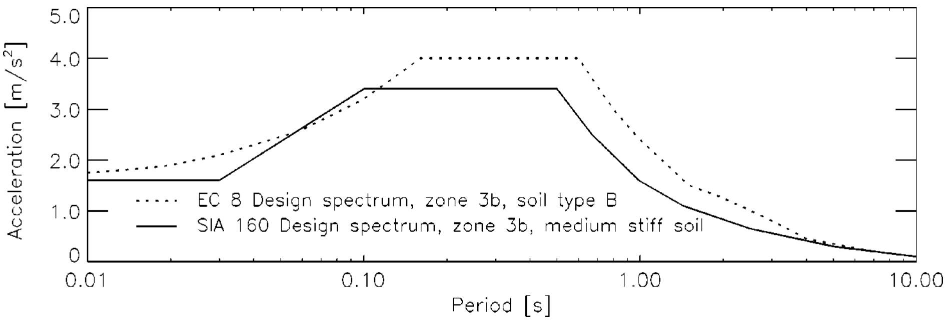


Figure 2. Elastic design spectra for 5 per cent damping: according to SIA 160, zone 3b, medium stiff soil (solid line), and according to Swiss application of EC 8, zone 3b, soil type B, (dotted line)

$W = 2{\cdot}78 + 3{\cdot}08 \cdot 7 = 24{\cdot}3$ MN. The total static equivalent force Q_{acc} is then determined as

$$Q_{\text{acc}} = \frac{a_{\text{h}}}{g} \cdot \frac{C_{\text{d}}}{K} \cdot (G_{\text{m}} + \Sigma\psi_{\text{acc}} \cdot Q_r) = 0{\cdot}135 \cdot \frac{0{\cdot}65}{2} \cdot 24{\cdot}3 \text{ MN} = 1{\cdot}07 \text{ MN} \tag{1}$$

where Q_{acc} is the total horizontal static design force, a_{h} the horizontal acceleration, m/s^2, see Figure 2, g the acceleration of gravity, m/s^2 and C_{d} the design factor, taken as 0·65, which takes into account the difference between design values and probable values of material strengths under earthquake conditions. C_{d} may be interpreted as

$$C_{\text{d}} = \frac{1}{\lambda_{\text{o}}\gamma_{\text{R}}} \tag{2}$$

where λ_{o} is the overstrength factor for steel reinforcement, taken as 1·2 to 1·25, and γ_{R} is the resistance factor taken as 1·2, K the deformation factor representing displacement ductility, $K = 2$ for SCI, G_{m} the mean value of self-weight of the structure and $\Sigma\psi_{\text{acc}} \cdot Q_{\text{r}}$ the sum of the other actions acting together with the earthquake.

The distribution of the static equivalent force over the height of the building is given by a triangular pattern,[5] linearly increasing from the base, without any extra force at the roof level.

This results in a shear force at the base of the wall $V_{\text{E}} = Q_{\text{acc}}$ and a bending moment at the base $M_{\text{E}} = 24{\cdot}0$ MNm. In general, additional forces would be considered due to torsion. Due to the plan symmetry of the studied building, according to the SIA 160 this is not needed in the studied case. This wall design is referred to as SIA8, indicating design code SIA and a height of eight storeys, see Table I which also contains the designs described subsequently. Details of the wall cross-section at the base of the wall is shown in Figure 3(a).

The flexural resistance M_{R}, including the normal force of 4·15 MN, assuming a strain at the compressive edge of 0·003, a rectangular concrete stress-block and ideal elasto-plastic behaviour of the reinforcing bars both in the web and in the boundary, equals 30·0 MNm and thereby meets the condition

$$M_{\text{R}} = 30{\cdot}0 \text{ MNm} \geqslant \gamma_{\text{R}} M_{\text{E}} = 1{\cdot}2 \cdot 24{\cdot}0 = 28{\cdot}8 \text{ MNm} \tag{3}$$

where γ_{R} is the resistance factor and M_{E} is the bending moment at the governing section resulting from the static equivalent force calculation.

For the shear design the minimum specified reinforcement ratio of $\rho_{\text{min}} = 0{\cdot}20$ per cent governs and results in horizontal bars D10/250 mm in both faces. The shear resistance V_{R} is calculated based only on the contribution from the shear reinforcement, and fulfills the following condition:

$$V_{\text{R}} = 3{\cdot}15 \text{ MN} \geqslant \gamma_{\text{R}} V_{\text{E}} = 1{\cdot}2 \cdot 1{\cdot}07 = 1{\cdot}28 \text{ MN} \tag{4}$$

Table I. Designation key for wall designs

Key	SIA4	SIA8	ECL4	ECH4	ECL8	ECH8
Design code	SIA 160	SIA 160	Eurocode 8	Eurocode 8	Eurocode 8	Eurocode 8
No. of stories	4	8	4	4	8	8
Ductility class			DCL	DCH	DCL	DCH

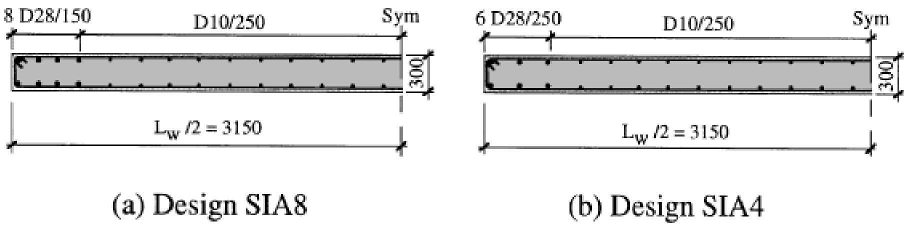


Figure 3. Details of cross-sections at the base of the walls designed according to SIA 160

For the four-storey version of the same building the most important design quantities as given above will be provided here. The fundamental period $T_1 = 0{\cdot}48$ s, which gives $a_h/g = 0{\cdot}34$ (plateau of the spectrum), $Q_{acc} = V_E = 1{\cdot}33$ MN, $M_E = 15{\cdot}7$ MNm. The wall cross section, referred to as design SIA4, is shown in Figure 3(b).

Design according to Eurocode 8

The basic approach in the Eurocode is similar to the one of SIA 160. Just like for SIA the most important steps in the design procedure will be demonstrated by using the eight-storey version of the example building.

In order to obtain the static equivalent force, the mass multiplied by the spectral acceleration is divided with a behaviour factor q derived based on ductility class, structural regularity, and prevailing failure mode. Distinction is made between different ductility classes, ranging from DCL over DCM to DCH, indicating low, medium and high ductility. In this study the ductility classes DCL with $q = 2$, and DCH with $q = 4$ are used. The corresponding designs will be referred to as ECL8 and ECH8.

For design ECH8 the wall thickness will be governed due to a requirement on the thickness of the unconfined regions due to lateral instability. The minimum thickness for this wall must be 420 mm, which is used in the design example, and will influence stiffness and period values. For design ECL8 the originally chosen wall thickness of 300 mm passes the requirement of lateral instability.

With wall thicknesses determined, the fundamental period of the structure may now be determined. Using the code formula of EC 8, which includes the wall thicknesses, the resulting periods are as follows (values from numerical model in parenthesis). For Design ECL8, $T_1 = 1{\cdot}31$ (1·30) s, and for design ECH8, $T_1 = 1{\cdot}10$ (1·08) s.

These periods are both within 2 percent of the respective results of numerical computations using only the structural walls (i.e. no nonstructural elements), and will therefore be used in the subsequent design procedure (EC 8 would allow the use of numerically computed periods if different from the code formula). Entering the design spectrum of the EC 8 Swiss National Application Document,[10] NAD, see Figure 2, zone 3b, soil type B, with the periods calculated above results in design acceleration ordinates as follows. For design ECL8, with $T_1 = 1{\cdot}31$ s, we obtain $S_D = 1{\cdot}5\ \mathrm{m/s^2}$. For design ECH8, with $T_1 = 1{\cdot}10$ s, we obtain $S_D = 2{\cdot}2\ \mathrm{m/s^2}$.

The seismic base shear force F_b according to EC 8, is obtained as follows:

$$F_b = \frac{S_D(T_1)}{g} \cdot \frac{1}{q} W. \tag{5}$$

Neglecting minor differences in calculation of the building weight compared to the SIA, for the sake of consistency, see also Wenk and Bachmann,[8] we obtain

For design ECL8: $F_b = \frac{1{\cdot}5}{10} \cdot \frac{1}{2} 24{\cdot}3 = 1{\cdot}82$ MN

For design ECH8: $F_b = \frac{2{\cdot}2}{10} \cdot \frac{1}{4} 24{\cdot}3 = 1{\cdot}34$ MN

The distribution of the static equivalent force over the height of the building in EC 8 is equivalent to the one for SIA 160 given above. Torsion is treated in several provisions (e.g. EC 8 1–2 3·2 and 3·3·2·4). However, acc. to EC 8 1–2, App. A4 torsional effects may be neglected in a symmetric plan case like the one studied. This exception is used here for simplicity and in consistency with the SIA 160 design. The design bending moments M_{Sd} at the base of the wall are obtained as follows. For design ECL8: $M_{Sd} = 40{\cdot}8$ MNm, and for ECH8: $M_{Sd} = 30{\cdot}0$ MNm. The flexural resistance is calculated assuming a strain at the compressive edge of 0·0035, a rectangular concrete stress-block and ideal elastoplastic behaviour of the reinforcing bars. Details of the wall cross-sections at the base are shown in Figures 4(a) and 4(b). In the figures, the length of the confined part of the cross-section is shown, with minimum length as prescribed in EC 8. In general, this length is ductility class dependent. For the walls studied here, the rule stipulating that at least 15% of the wall length be confined is governing. Further details on hoops and cross ties are provided in EC 8.

The design shear forces V_{Sd} for the walls are determined as follows:

$$V_{Sd} = \varepsilon \cdot V'_{Sd} \tag{6}$$

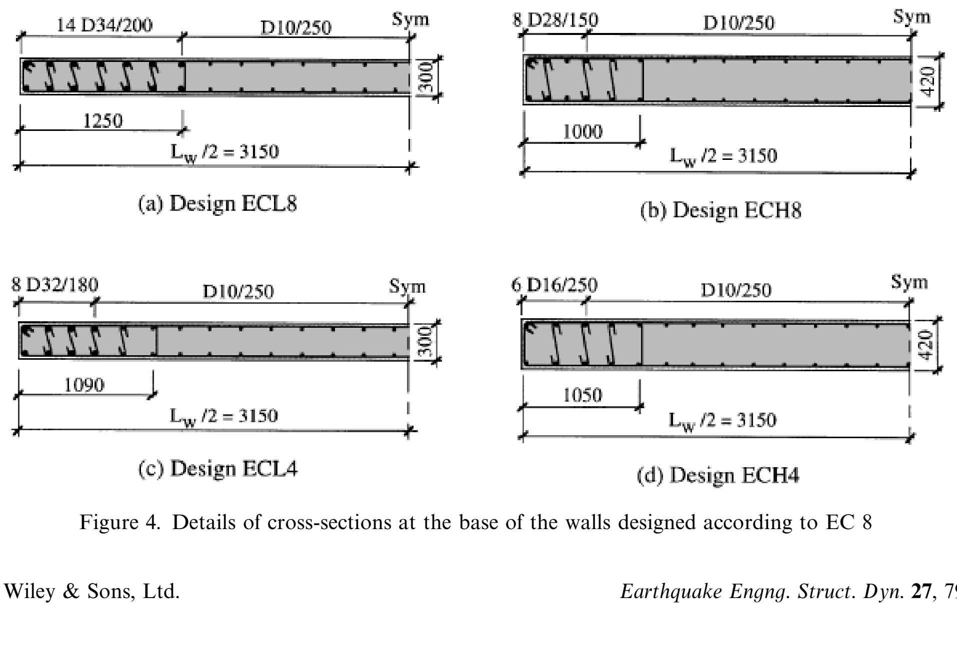


Figure 4. Details of cross-sections at the base of the walls designed according to EC 8

where $V'_{\rm Sd}$ is the shear force obtained from the static equivalent forces and ε the amplification factor obtained as

$$\text{For DCL: } \varepsilon = 1{\cdot}3 \tag{7a}$$

$$\text{For DCH: } \varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{\rm Rd}}{q} \cdot \frac{M_{\rm Rd}}{M_{\rm Sd}}\right)^2 + 0{\cdot}1\left(\frac{S_{\rm e}(T_C)}{S_{\rm e}(T_1)}\right)^2} \leqslant q \tag{7b}$$

in which q in the behaviour factor, $M_{\rm Sd}$ the design bending moment at the base of the wall, MNm, $M_{\rm Rd}$ the design flexural resistance at the base of the wall, MNm, $\gamma_{\rm Rd}$ the overstrength ratio of steel; may be taken as 1·25 for DCH and 1·15 for DCM, T_1 the fundamental period of building in the direction of the wall, seconds, T_C the upper limit period of constant spectral acceleration branch, seconds and $S_{\rm e}(T)$ the ordinate of the elastic response spectrum.

In our example, we obtain for design ECL8: $\varepsilon = 1{\cdot}3$, and using equation (7b) for design ECH8:

$$\varepsilon = 4 \cdot \sqrt{\left(\frac{1{\cdot}25}{4} \cdot \frac{31{\cdot}6}{30{\cdot}0}\right)^2 + 0{\cdot}1\left(\frac{4{\cdot}0}{2{\cdot}2}\right)^2} = 2{\cdot}65$$

Thus, using equation (6) the design shear forces at the wall base are obtained as follows. For design ECL8: $V_{\rm Sd} = 1{\cdot}3 \cdot 1{\cdot}82 = 2{\cdot}37$ MN, and for ECH8: $V_{\rm Sd} = 2{\cdot}65 \cdot 1{\cdot}34 = 3{\cdot}55$ MN.

For the determination of the shear resistance, distinction is made between regions inside and outside the critical region of the wall. The height of the critical region, $h_{\rm cr}$, is determined as follows:

$$h_{\rm cr} = \max\{l_{\rm w}, H_{\rm w}/6\} \tag{8}$$

To check the diagonal tension failure of the web the shear resistance $V_{\rm Rd3}$ is obtained as follows.

$$V_{\rm Rd3} = V_{\rm CD} + V_{\rm wd} \tag{9}$$

in which $V_{\rm CD}$ is the concrete contribution, for which distinction is made for the critical region within which no contribution for normal force may be accounted for[4] in ductility class DCH and $V_{\rm wd}$ the reinforcement contribution.

For design ECL8 minimum reinforcement with D10/250 is sufficient over the entire height of the wall, and at the wall base the following condition is fulfilled:

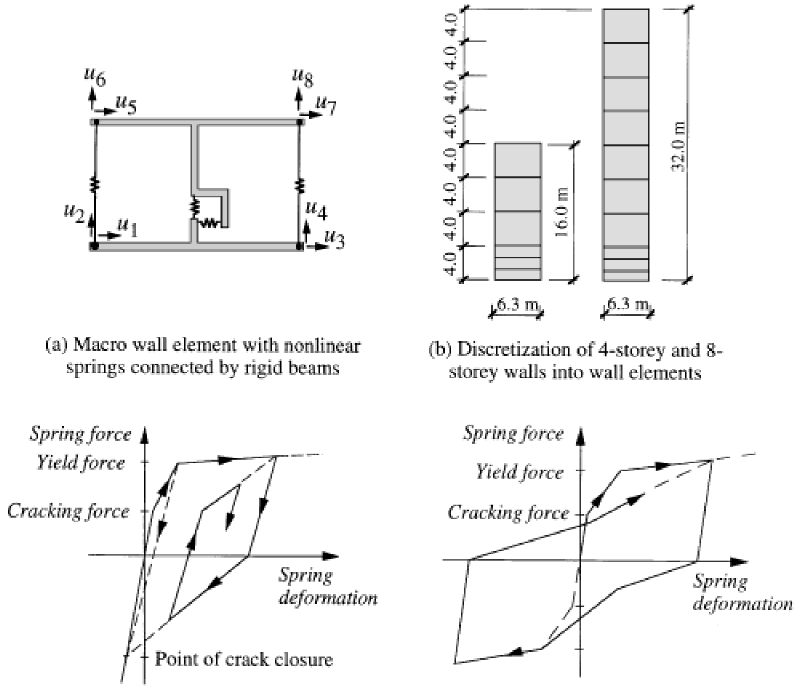
$$V_{\rm Rd3} = V_{\rm CD} + V_{\rm wd} = 1{\cdot}29 + 1{\cdot}24 = 2{\cdot}53 \text{ MN} \geqslant \varepsilon V'_{\rm Sd} = 1{\cdot}3 \cdot 1{\cdot}82 = 2{\cdot}37 \text{ MN} \tag{11}$$

In addition checks on diagonal compression failure and sliding shear are to be carried out. For design ECH8 the critical region requires D12/170 which is reduced in three steps over the height of the wall to reach a minimum of D10/180 above the sixth storey.

For the four-storey designs ECL4 and ECH4 the important design quantities are given correspondingly as follows: For ECL4: $T_1 = 0{\cdot}52$ s (0·48 s, using numerical model), $F_{\rm b} = 2{\cdot}4$ MN, $M_{\rm Sd} = 28{\cdot}5$ MNm, for design ECH4: $T_1 = 0{\cdot}44$ s (0·32 s, using numerical model), $F_{\rm b} = 1{\cdot}2$ MN, $M_{\rm Sd} = 14{\cdot}2$ MNm. For these walls, the code formula fundamental periods do not show as good agreement with the numerical model, however, all these period values are within the plateau region of the design spectrum and thus result in the same spectral values. Details of the wall cross-sections at the base are shown in Figures 4(c) and 4(d).

NUMERICAL MODELLING AND ANALYSIS

In order to study the behaviour of the walls designed in the previous section, the walls were modelled numerically and subjected to a dynamic analysis. Due to symmetry, only one wall (out of two) in the short direction was analysed, and consequently the masses used in the analysis corresponded to the floor masses divided in two.



(a) Macro wall element with nonlinear springs connected by rigid beams

(b) Discretization of 4-storey and 8-storey walls into wall elements

(c) Hysteretic rules for outer vertical (flexural) springs

(d) Hysteretic rules for horizontal (shear) spring

Figure 5. Numerical model; macro wall element, discretization, and hysteretic rules

A numerical model[9,11] developed particularly for the global analysis of reinforced concrete structural walls, was used. Similar models have been developed by other authors.[13,14] This model which is made up of non-linear springs connected by rigid beams, has proven capable of simulating the most important characteristics of the non-linear behaviour of such walls, such as large tensile strains, shifting of the neutral axis, and typical hysteretic behaviour in flexure and shear. The model is shown in the form of a wall element in Figure 5(a), and the discretization of the eight-storey wall is shown in Figure 5(b).

In each element, the three vertical springs together provide for realistic non-linear axial and flexural behaviour. These springs are calibrated according to the behaviour of the walls' cross-section whereby a moment curvature diagram resulting from a fibre cross-section analysis is used. In the fibre cross-section analysis the effect of confinement on the concrete compressive behaviour is taken into consideration, which effect is thereby transferred to the wall model. The horizontal spring simulates non-linear shear behaviour. The general hysteretic model for the outer vertical springs is shown in Figure 5(c) and for the horizontal spring in Figure 5(d). Both hysteretic models were improved and are now trilinear. The central vertical spring is active only in compression.[9]

As seen in Figure 5(b) one wall element is used for each storey except for the first storey, where the flexural plastic hinge will be located. This storey is discretized into three elements. The storey masses corresponding to half the structure were modelled by two lumped mass elements per floor level (one at each wall element node). The major damping originates from the hysteretic damping in the non-linear wall elements. In addition, 2 per cent Rayleigh damping is provided determined at each analyzed wall's first and third mode of vibration. The non-linear wall element was coded as a user defined element and added to the general finite element program ABAQUS.[12]

Three ground acceleration time histories are used in this study, shown in Figures 6–8. The first two are artificially generated and are compatible with the elastic design spectra used in the respective designs. The history in Figure 6(a) is compatible with the design spectrum of SIA 160, zone 3b and medium stiff soil shown in Figure 6(b) in conjunction with the response spectrum of the ground motion. The history in Figure 7(a) is compatible with the design spectrum of EC 8, zone 3b and soil type B, shown in Figure 7(b) with the response spectrum of the ground motion. Generated ground motions which are design spectrum compatible, tend not to be fully realistic, and therefore a third recorded ground motion is used in this study as a complement. This is the N–S component recorded at Tolmezzo, of the 1976 Friuli earthquake of Northern Italy. The history is shown in Figure 8(a) and the response spectrum in Figure 8(b).

One wall only, running in the short direction in plan, was analysed for each design. In a first analysis step gravity loads were applied. These gravity loads were based on the weight of the wall and in addition weight, dead and live loads for the tributary slab area of each floor. Elastic behaviour was assumed during this step. In a second analysis step a non-linear dynamic time history analysis was carried out using either one of the acceleration time histories of Figure 6–8 as horizontal component in the direction of the wall. Implicit integration according to the Hilber–Hughes–Taylor scheme[12] with time increments of 0·01 s was used. Full Newton–Raphson iteration was employed with a maximum unbalanced element force of 2 per cent of maximum element forces allowed without iteration. The gravity loads remained active during this step. The analysis duration for all ground motions was 12 s.

The SIA4 and SIA8 designs were analysed with the ground motion of Figure 6(a) as input, in order to relate the behaviour directly to the design. The ECL4, ECH4, ECL8 and ECH8 designs were similarly analyzed with the ground motion of Figure 7(a) as input. Finally, in order to study the behaviour of all designs in a relative manner, the recorded ground motion of Figure 8(a) was used.

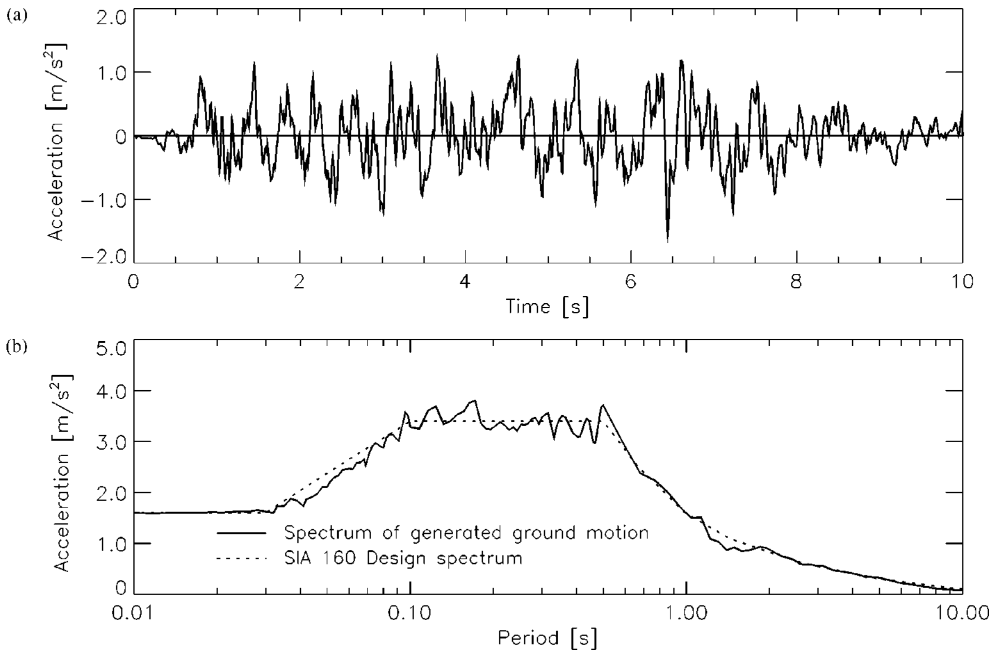


Figure 6. Artificially generated ground motion compatible to design spectrum of SIA 160: acceleration time history (a), response spectrum for 5 per cent damping of ground motion (solid line) and elastic design spectrum for 5 per cent damping, zone 3b, medium stiff soil (b)

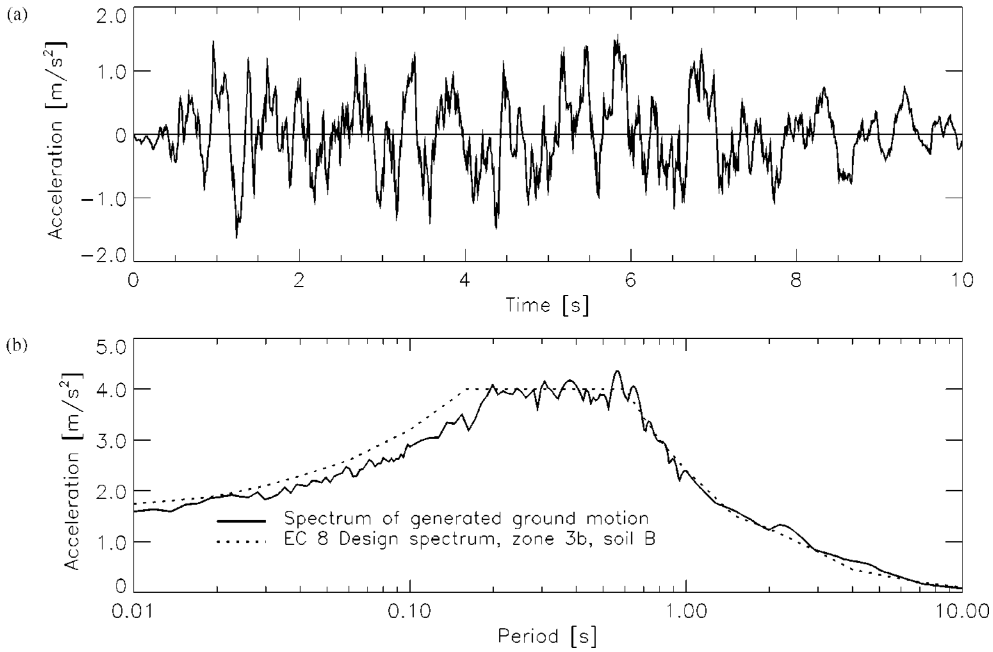


Figure 7. Artificially generated ground motion compatible to design spectrum of EC 8: acceleration time history (a), response spectrum for 5 per cent damping of ground motion (solid line) and elastic design spectrum for 5 per cent damping, zone 3b, soil type B (b)

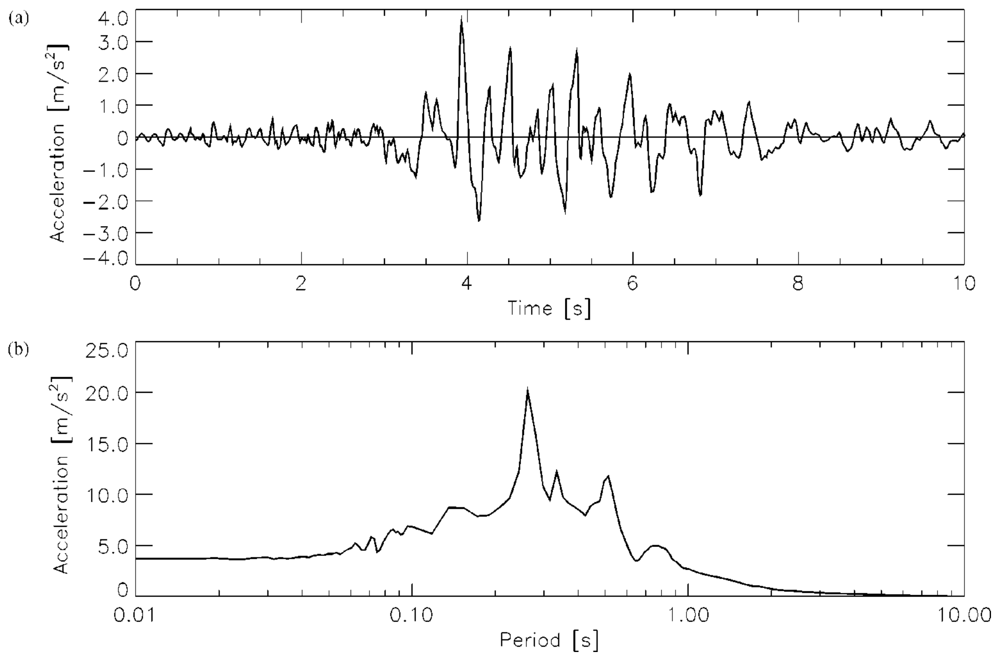


Figure 8. Ground motion of the 1976 Friuli earthquake recorded at Tolmezzo: N–S component of acceleration time history (a), response spectrum for 5 per cent damping of N–S component (b)

DISCUSSION OF NUMERICAL RESULTS

In this section results of the numerical analyses introduced in the previous section are discussed. The discussion will focus on the behaviour of the single wall. Figures 9–13 each show the following three response quantities as a function of time: roof displacement, shear force at wall base, and bending moment at wall base.

Figure 9 shows the results for the two designs SIA4 and SIA8, both subjected to the ground motion of Figure 6(a) which is compatible with the SIA design spectrum. The different fundamental period of vibration of the four-storey and the eight-storey wall is clearly seen in Figure 9(a). Higher mode effects are also seen for the base shear forces in Figure 9(b), causing a demand that is in the vicinity of the nominal shear resistance of 3·15 MN for both walls, which however only takes the shear reinforcement into account. In Figure 9(c) flexural overstrength is seen particularly for the four-storey wall, with strengths around 25 MNm, to be compared with the nominal design strength of around 15 MNm, assuming only design values for material and no strain hardening. In contrast, the numerical model uses realistic strength values and strain hardening.

Figures 10 and 11 show the corresponding quantities for the Eurocode based four-storey designs ECL4 and ECH4 (Figure 10) and eight-storey designs ECL8 and ECH8 (Figure 11), all subjected to the ground motion of Figure 7(a) which is compatible with the Eurocode design spectrum. Effects from higher modes are also seen for all these four designs, particularly for the base shear forces. Especially, the designs for low ductility, using a shear magnification factor of only 1·3, obtain large shear forces exceeding the resistance in a manner not expected in a low ductility design, indicating that this factor may be on the low side. The two designs for high ductility display clear flexural overstrength, which they are however detailed for.

Figures 12 and 13 show the above quantities for all designs subjected to the ground motion of Figure 8(a) which is the record at Tolmezzo of the Friuli earthquake. Figure 12 shows the results for all four-storey walls, and Figure 13 shows the results for all eight-storey walls. The higher spectral values of this ground motion in

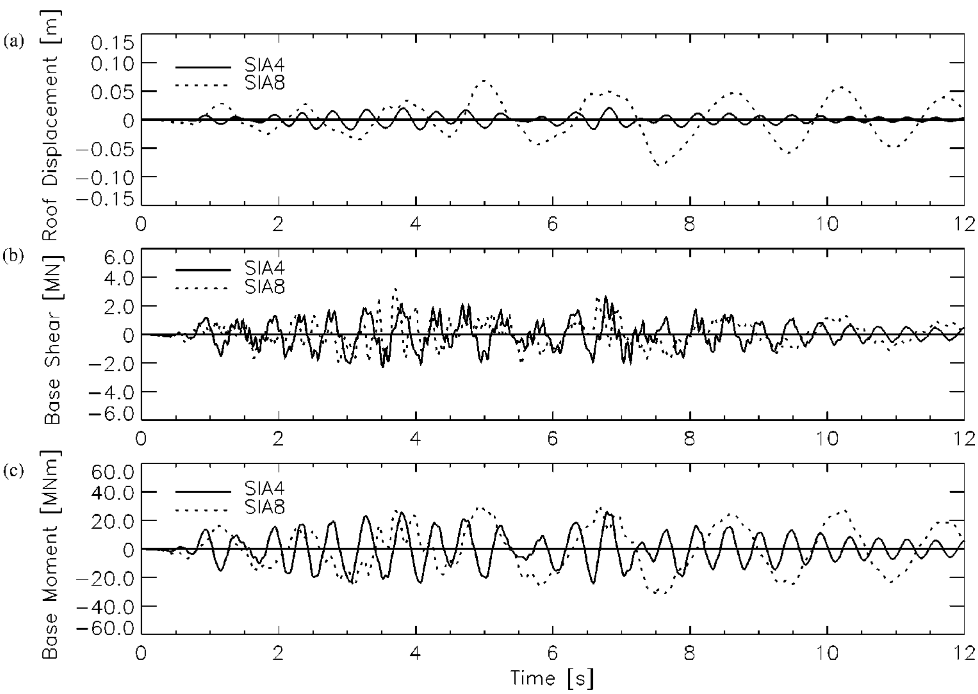


Figure 9. Time-history response quantities of designs SIA4 and SIA8, subjected to SIA 160 compatible ground motion

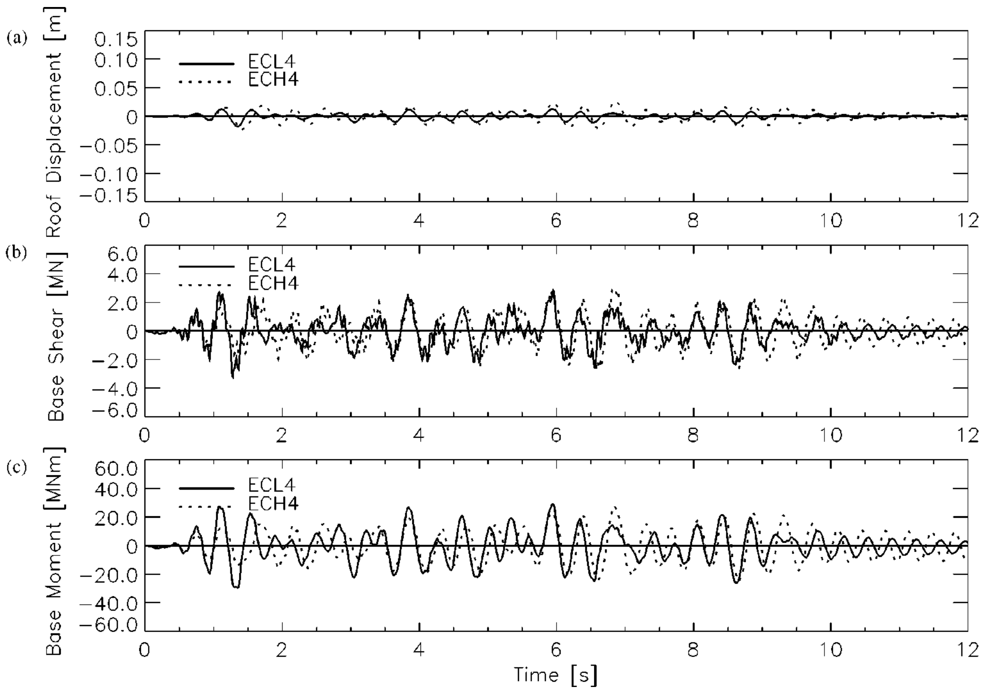


Figure 10. Time-history response quantities of designs ECL4 and ECH4, subjected to EC 8 compatible ground motion

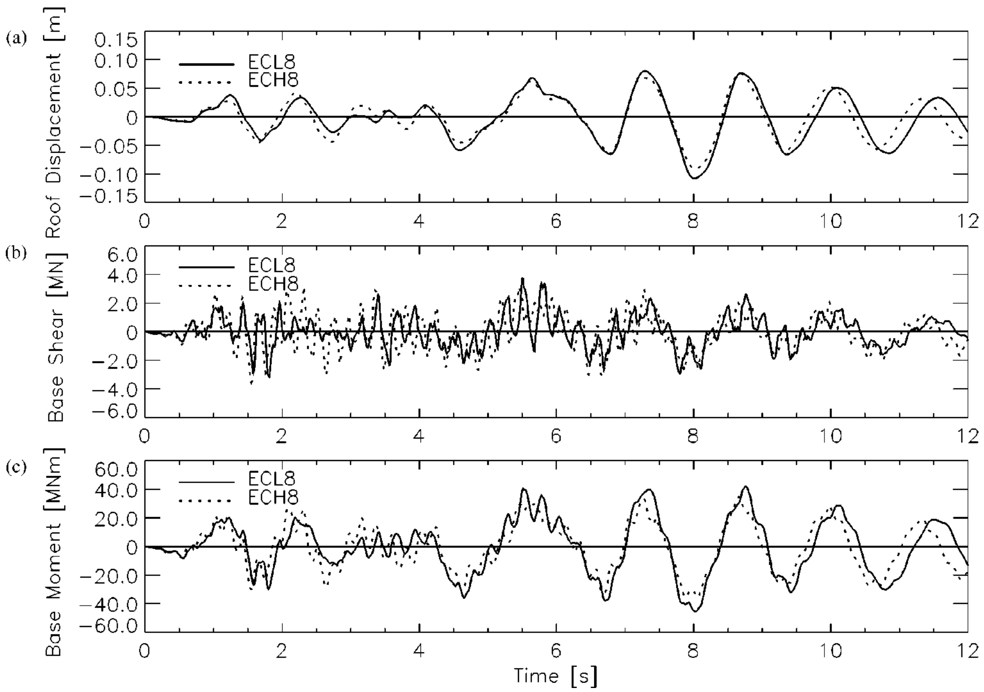


Figure 11. Time-history response quantities of designs ECL8 and ECH8, subjected to EC 8 compatible ground motion

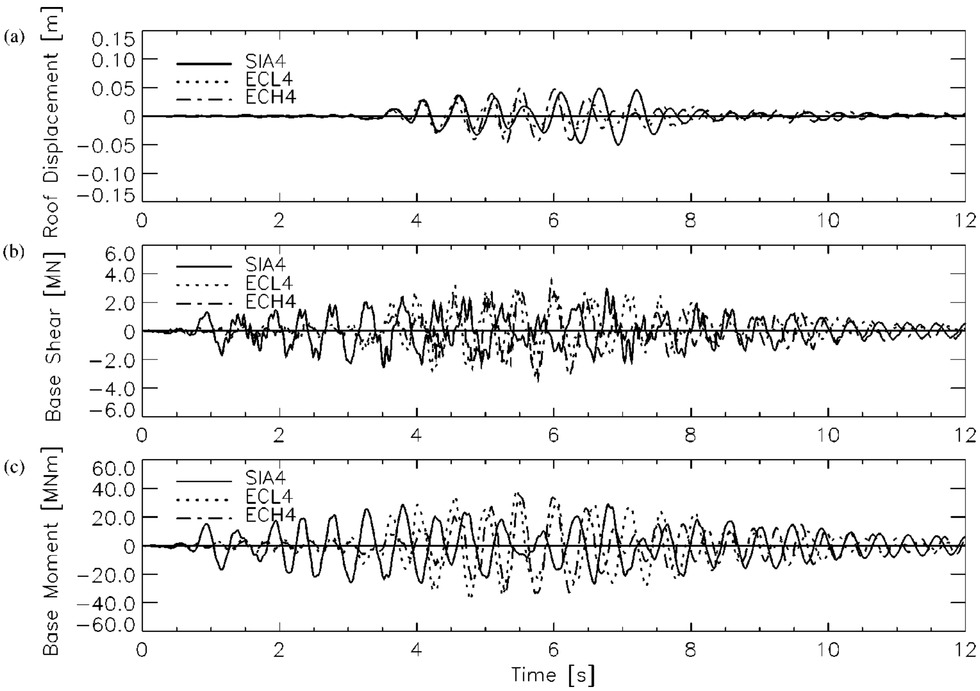


Figure 12. Time-history response quantities of designs SIA4, ECL4, and ECH4, subjected to ground motion recorded at Tolmezzo

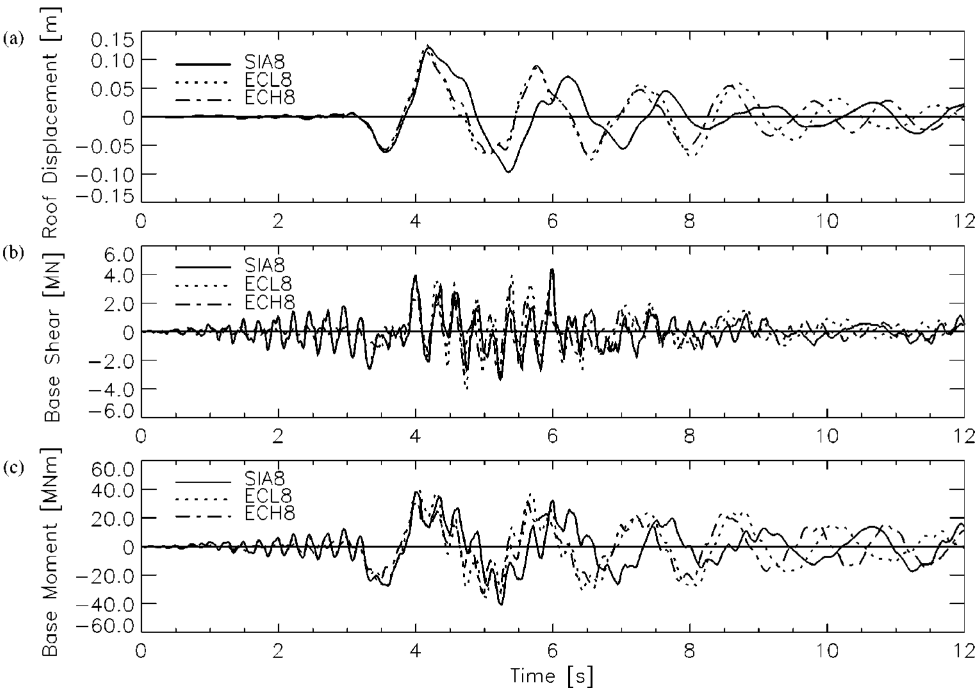


Figure 13. Time-history response quantities of designs SIA8, ECL8, and ECH8, subjected to ground motion recorded at Tolmezzo

Table II. Maximum curvature ductility demand, and maximum Interstorey drift

Maximum curvature ductility demand, 1st storey			
Design	SIA	EC 8	Tolmezzo
SIA4	3·0		4·1
SIA8	3·2		5·7
ECL4		3·3	2·5
ECH4		5·8	7·1
ECL8		3·0	2·8
ECH8		4·2	4·8
Maximum interstorey drift (m) (storey no.)			
Design	SIA	EC 8	Tolmezzo
SIA4	0·007 (4)		0·014 (4)
SIA8	0·013 (8)		0·023 (8)
ECL4		0·005 (4)	0·086 (3)
ECH4		0·008 (4)	0·017 (4)
ECL8		0·019 (8)	0·023 (8)
ECH8		0·015 (8)	0·020 (8)

general result in somewhat larger response quantities. In particular for base shear, all designs display a higher demand than for the ground motions compatible with their respective design codes.

Table II summarizes the maximum curvature ductility demand and maximum inter-storey storey drift values obtained for each design and for each ground motion. For the curvature demand, it is seen that although a more slender wall in general is expected to display somewhat higher curvature ductility than a squat wall, the effect of the axial load counteracts this trend. The reduced axial load for the four-storey walls compared with the eight-storey walls in some cases result in larger ductility values.

For the storey drift only the values for the eight storey walls will be discussed, since the drift for the four-storey walls is small. For design SIA8 the drift is 0·013 m for SIA and 0·023 m for Tolmezzo which correspond to relative inter-storey drifts of 0·0033 and 0·0058, the first of which is within the SIA 160 limit of 0·005 for general multi-storey buildings. The two designs ECL8 and ECH8 obtained for the EC 8 compatible ground motion inter-storey drifts of 0·019 and 0·015 m, corresponding to relative inter-storey drifts of 0·0048 and 0·0038, to be compared with an EC 8 limit of 0·006 for serviceability states. An additional calculation was performed for the ECL8 design with the ground motion scaled down by a factor of 2·0 to achieve a 'serviceability action'. This resulted in a maximum roof displacement of 0·07 m, and a maximum relative inter-storey drift of 0·0025. The drift values for these designs subjected to the ground motion recorded at Tolmezzo are slightly larger.

As a complement to Figures 9–13, the distribution over the height of the wall of shear forces and bending moments is shown in Figure 14 for all walls. In the figure, the demand is shown for each storey, and in the first storey for each of the three elements, as the maximum values obtained during the non-linear time-history analysis. For each wall two demand curves are shown; one resulting from the ground motion compatible with the code specific response spectrum, and one resulting from the ground motion recorded at Tolmezzo. For all walls, the demands are shown in conjunction with the distribution of shear force/bending moment resulting from the equivalent static force, and with the resistance of the wall as designed. It should be noted that the resistance in both the case of shear force and bending moment is based upon design values for strength, and does not take e.g. strain hardening into account. This explains why the demand may exceed the resistance in a number of instances, which are marked in different patterns explained in the figure.

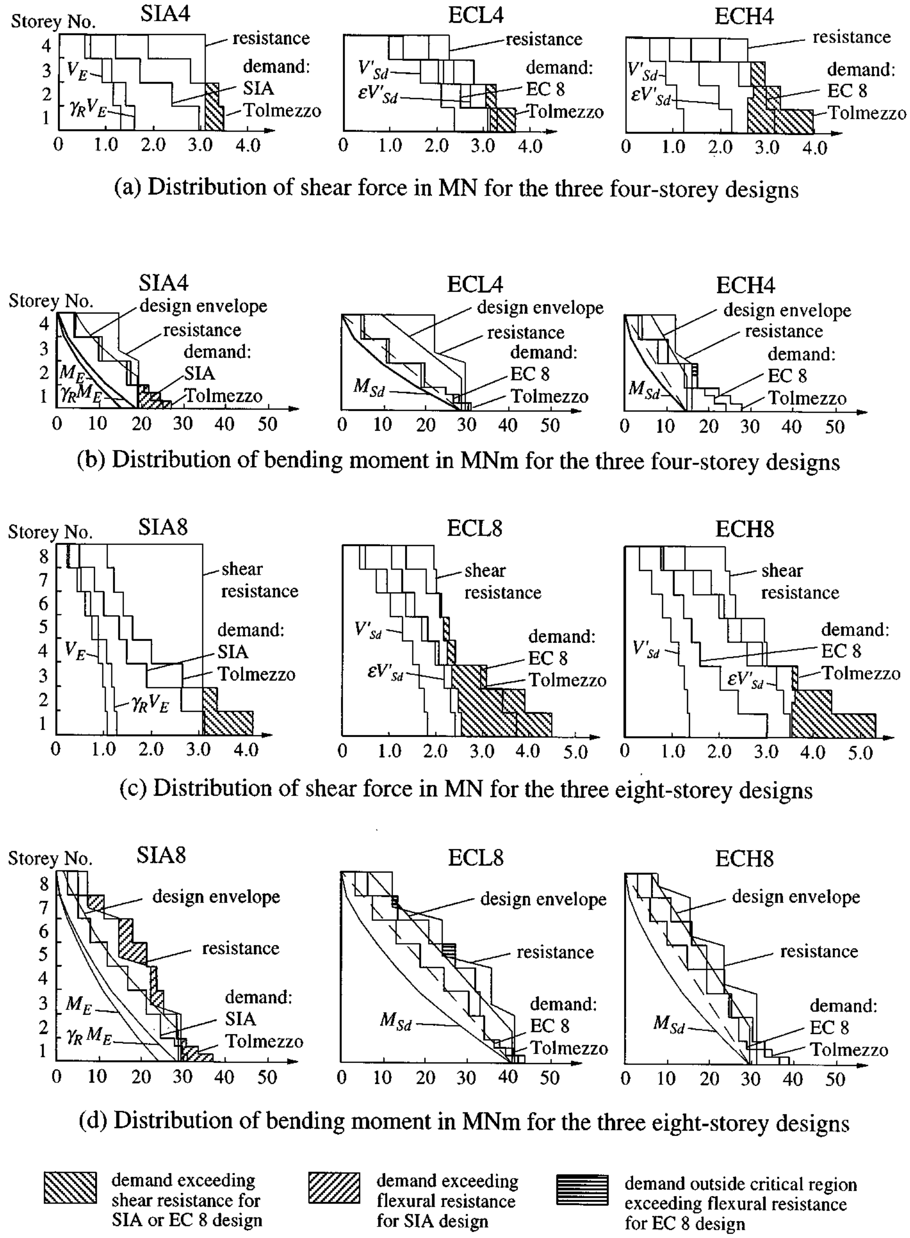


Figure 14. Distribution over wall height of shear force and bending moment demand resulting from maximum values obtained during non-linear time-history analysis, in conjunction with design forces and resistance

For all designs, at least the shear demand from the Tolmezzo ground motion exceeded the resistance according to code. Large shear demands in relation to resistance are observed for the design ECL8, attributed to the low shear magnification factor of 1·3. From Figure 14(d) is also seen that for the design SIA8 the flexural demand exceeds the resistance in most upper stories, which is virtually not the case for the designs ECL8 and ECH8. This is explained by their linear design envelope above the critical region, which allows for the effect of higher modes more than the SIA design envelope which follows the moment curve. For the EC 8

designed walls demand exceeding flexural resistance is only marked when it occurs in regions outside the critical region, inside of which flexural plastification is tolerated and accounted for by structural detailing.

However, here as well as for other result quantities, different ground motion records should be used in order to reach conclusive results.

CONCLUSIONS

The studies presented in this paper allow for the following conclusions.

(1) The Eurocode 8 formula for the fundamental frequency in this study proved to agree better with a finite element model than the formula of SIA 160. This was particularly the case for the eight-storey walls.
(2) The overestimation of the fundamental frequency by the formula in SIA 160 results in an overdesigned structural wall, which will not reach flexural yielding when subjected to a ground motion compatible to the used design spectrum despite an assumed displacement ductility factor of 2.
(3) The overall behaviour of the EC 8 designed walls with low ductility was similar to the SIA designed wall. The high ductility class of EC 8 has no counterpart in SIA 160.
(4) For horizontally long walls, as used here, the requirement on wall thickness in order to avoid lateral buckling in EC 8 will result in unreasonably thick wall webs (thicker than the boundary elements would be required to be). This requirement should be reviewed for long walls.
(5) The introduction of a magnification factor for shear in EC 8 represents an improvement compared to SIA. However, the relatively low magnification factor of 1·3, used in EC 8 for walls of 'ductility class low', DCL, may, just like in the case of SIA 160 where no magnification factor is used, result in a shear failure.
(6) The linear design envelope for flexural resistance in EC 8 is more conservative than the envelope of SIA following the moment curve, and in these examples largely led to avoidance of flexural yielding in the upper stories. Such yielding did occur for the SIA designed eight-storey wall, mainly due to higher mode effects.
(7) The studies of this paper should be complemented with studies employing different numerical models, different ground motions, and using buildings of different configurations, among others with non-symmetric plans so as to obtain an evaluation of provisions for torsion.

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